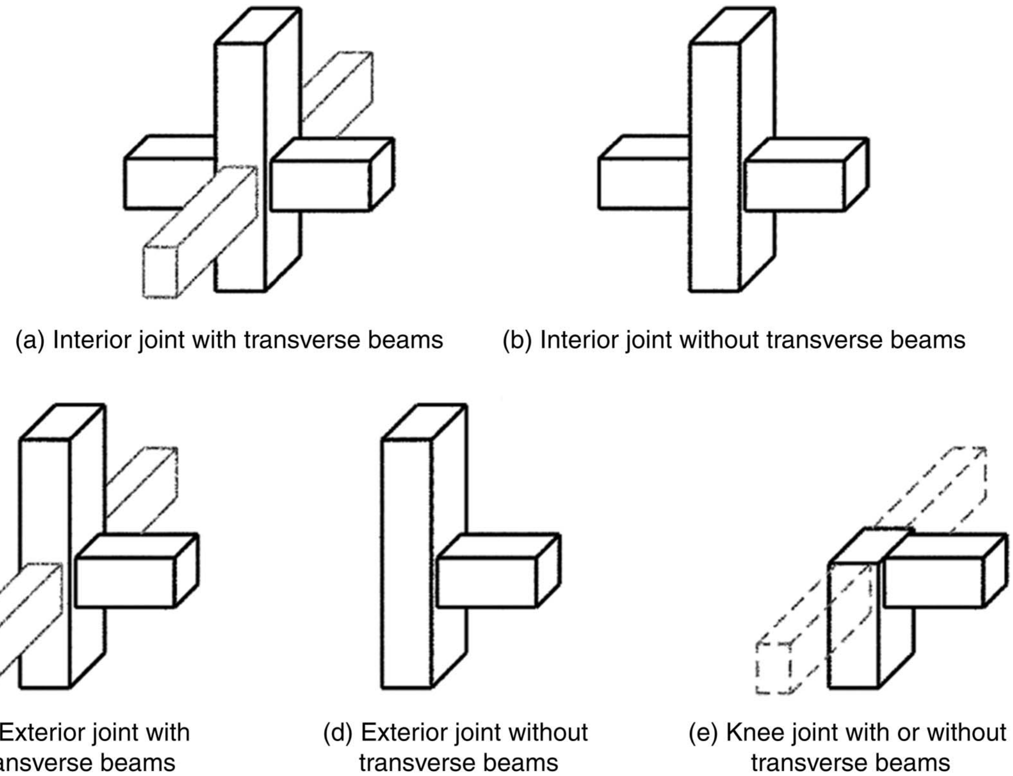

Figure 10-3. Joint Classification (for Response in the Plane of the Page)

specified in Section 10.4.2.1 for reinforced concrete beam–column moment frames. In addition to potential failure modes described in Section 10.4.2.1, the analysis model shall consider potential failure of tendon anchorages.

The analysis procedures described in Chapter 7 apply to frames with post-tensioned beams satisfying the following conditions:

1. The average prestress f_{pc} calculated for an area equal to the product of the shortest and the perpendicular cross-sectional dimensions of the beam does not exceed the greater of 750 lb/in.^{2} (5 MPa) or $f'_{cL}/12$ at locations of nonlinear action;
2. Prestressing tendons do not provide more than one-quarter of the strength at the joint face for both positive and negative moments; and
3. Anchorages for tendons are demonstrated to have performed satisfactorily for seismic forces in compliance with ACI 318 requirements. These anchorages shall occur outside hinging areas or joints, except in existing components where experimental evidence demonstrates that the connection meets the Performance Objectives under design loadings.

Alternative procedures shall be used where these conditions are not satisfied.

10.4.3.2 Stiffness of Post-tensioned Concrete Beam–Column Moment Frames

10.4.3.2.1 Linear Static and Dynamic Procedures. Beams shall be modeled considering flexural and shear stiffnesses, including the effect of the slab acting as a flange in monolithic and composite construction. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Refer to Section 10.3.1.2 for effective stiffness computations. Refer to Section 10.4.2.2.1 for modeling of joint stiffness.

10.4.3.2.2 Nonlinear Static Procedure. Nonlinear load–deformation relations shall comply with Section 10.3.1.2 and reinforced concrete frame requirements of Section 10.4.2.2.2.

Values of the generalized deformation at points B, C, and D in Fig. 10-1 shall be derived either from experiments or from approved rational analyses, considering the interactions among flexure, axial load, and shear. Alternatively, where the generalized deformation is taken as rotation in the flexural plastic hinge zone and the three conditions of Section 10.4.3.1 are satisfied, beam plastic hinge rotation capacities shall be permitted to be as defined in Table 10-7. Columns and joints shall be modeled as described in Section 10.4.2.2.

10.4.3.2.3 Nonlinear Dynamic Procedure. For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. Fig. 10-1 shall be taken to represent the envelope relation for the analysis. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics as influenced by prestressing.

10.4.3.3 Strength of Post-tensioned Concrete Beam–Column Moment Frames. Component strengths shall be computed according to the general requirements of Section 10.3.2 and additional requirements of Section 10.4.2.3. Effects of prestressing on strength shall be considered.

For deformation-controlled actions, prestress shall be assumed effective to determine the maximum actions that can be developed in association with nonlinear response of the frame. For force-controlled actions, the effects on strength of prestress loss shall be considered as a design condition, where such losses are possible under design–load combinations including inelastic deformation reversals.

10.4.3.4 Acceptance Criteria for Post-tensioned Concrete Beam–Column Moment Frames. Acceptance criteria for post-tensioned concrete beam–column moment frames shall follow the criteria for reinforced concrete beam–column frames specified in Section 10.4.2.4.

Modeling parameters and acceptance criteria shall be based on Tables 10-7 through 10-10, 10-13, and 10-14.

10.4.3.5 Retrofit Measures for Post-tensioned Concrete Beam–Column Moment Frames. Seismic retrofit measures for post-tensioned concrete beam–column moment frames shall meet the requirements of Section 10.3.7 and other provisions of this standard.

10.4.4 Slab–Column Moment Frames

10.4.4.1 General. The analytical model for a slab–column frame element shall represent strength, stiffness, and deformation capacity of slabs, columns, slab–column connections, and other components of the frame. The connection between the columns and foundation shall be modeled based on the details of the column–foundation connection and rigidity of the foundation–soil system. Potential failure in flexure, shear, shear-moment transfer (punching shear), and reinforcement development at any section along the component length shall be considered. The effects of changes in cross section, slab openings, and interaction with structural and nonstructural components shall be considered.

An analytical model of the slab–column frame based on any of the following approaches shall be permitted to be used:

1. Effective beam width model: Columns and slabs are represented by line elements rigidly interconnected at the slab–column connection, where the slab width included in the model is adjusted to account for flexibility of the slab–column connection;
2. Equivalent frame model: Columns and slabs are represented by line elements, and stiffness of column or slab elements is adjusted to account for flexibility of the slab–column connection; and
3. Finite element model: Columns are represented by line elements and the slab by plate-bending elements.

10.4.4.2 Stiffness of Slab–Column Moment Frames

10.4.4.2.1 Linear Static and Dynamic Procedures. Slabs shall be modeled considering flexural, shear, and torsional (in the slab adjacent to the column) stiffnesses. Columns shall be modeled considering flexural, shear, and axial stiffnesses. Slab–column connections shall be modeled as stiff or rigid components. Although effective component stiffnesses shall be determined according to the general principles of Section 10.3.1.2, adjustments shall be permitted based on experimental evidence.

10.4.4.2.2 Nonlinear Static Procedure. Nonlinear load–deformation relations shall comply with the requirements of Section 10.3.1.2. Nonlinear modeling parameters for slab–column connections are provided in Table 10-15.

Nonlinear static models shall be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends.

Idealized load–deformation relations shall be modeled using the generalized relation shown in Fig. 10-1. The overall load–deformation relation shall be established so that the maximum resistance is consistent with the strength specifications of Sections 10.3.2 and 10.4.4.3. For columns, the generalized deformation shown in Fig. 10-1 is flexural plastic hinge rotation with parameters as defined in Tables 10-8 and 10-9. For slabs and slab–column connections, the generalized deformation shown in Fig. 10-1 is plastic rotation with parameters as defined in Table 10-15. Different relations shall be permitted where verified by experimentally obtained cyclic response relations of slab–column subassemblies.

10.4.4.2.3 Nonlinear Dynamic Procedure. The requirements of Sections 10.3.2 and 10.4.2.2.3 for reinforced concrete beam–column moment frames shall apply to slab–column moment frames.

10.4.4.3 Strength of Slab–Column Moment Frames. Component strengths shall be computed according to the general requirements of Section 10.4.2, as modified in this section. For columns, evaluation of shear strength according to Section 10.4.2.3 shall be permitted to be used.

The flexural strength of a slab to resist moment caused by lateral deformations shall be calculated as $M_{SICSE} - M_{UD,CS}$. Slab–column connections shall be investigated for potential failure in shear and moment transfer, considering the combined action of flexure, shear, and torsion acting in the slab at the connection with the column.

For interior connections without transverse beams and exterior connections with moment about an axis perpendicular to the slab edge, the shear and moment transfer strength, or the “torsional” element strength, shall be permitted to be calculated as the minimum of

1. Strength calculated considering eccentricity of shear on a slab-critical section because of combined shear and moment in accordance with ACI 318; and
2. Moment transfer strength equal to $\Sigma M_{SIE}/\gamma_f$ where ΣM_{SIE} is the sum of positive and negative flexural strengths of a section of slab between lines that are two and one-half slab or drop panel thicknesses ($2.5h$) outside opposite faces of the column or capital; γ_f is the fraction of the moment resisted by flexure, per ACI 318.

For moment about an axis parallel to slab edge at exterior connections without transverse beams, where the shear on the slab critical section caused by gravity loads does not exceed $0.75V_{CPunE}$ or the shear at a corner support does not exceed $0.5V_{CPunE}$, the moment transfer strength shall be permitted to be taken as equal to the flexural strength of a section of slab between lines that are a distance c_1 outside opposite faces of the column or capital.

10.4.4.4 Acceptance Criteria for Slab–Column Moment Frames

10.4.4.4.1 Linear Static and Dynamic Procedures. Component actions shall be classified as being deformation controlled or force controlled, as defined in Section 10.3.2.1. In primary components, deformation-controlled actions shall be restricted to flexure in slabs and columns, and shear and moment transfer in slab–column connections. In secondary components, deformation-controlled actions are permitted in shear and reinforcement development (Table 10-16). All other actions shall be classified as force controlled.

Design actions on components shall be determined as prescribed in Chapter 7. Where the calculated DCR values exceed unity, the following design actions shall be determined using limit analysis principles as prescribed in Chapter 7:

Table 10-15. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Two-Way Slabs and Slab–Column Connections

		Modeling Parameters[a]			Acceptance Criteria[a]		
					Plastic Rotation Angle (radians)		
					Performance Level		
		Plastic Rotation Angle (radians)		Residual Strength Ratio		Secondary	
Conditions		***a***	***b***	***c***	**IO**	**LS**	**CP**
Condition i. Reinforced concrete slab–column connections[b]							
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]						
0	Yes	0.035	0.05	0.2	0.01	0.035	0.05
0.2	Yes	0.03	0.04	0.2	0.01	0.03	0.04
0.4	Yes	0.02	0.03	0.2	0	0.02	0.03
≥0.6	Yes	0	0.02	0	0	0	0.02
0	No	0.025	0.025	0	0.01	0.02	0.025
0.2	No	0.02	0.02	0	0.01	0.015	0.02
0.4	No	0.01	0.01	0	0	0.008	0.01
0.6	No	0	0	0	0	0	0
>0.6	No	0	0	0	—[e]	—[e]	—[e]
Condition ii. Post-tensioned slab–column connections[b]							
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]						
0	Yes	0.035	0.05	0.4	0.01	0.035	0.05
0.6	Yes	0.005	0.03	0.2	0	0.025	0.03
>0.6	Yes	0	0.02	0.2	0	0.015	0.02
0	No	0.025	0.025	0	0.01	0.02	0.025
0.6	No	0	0	0	0	0	0
>0.6	No	0	0	0	—[e]	—[e]	—[e]
Condition iii. Slabs controlled by inadequate development or splicing along the span[b]							
		0	0.02	0	0	0.01	0.02
Condition iv. Slabs controlled by inadequate embedment into slab–column joint[b]							
		0.015	0.03	0.2	0.01	0.02	0.03

[a] Values between those listed in the table shall be determined by linear interpolation.
[b] Where more than one of conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.
[c] V_g is the gravity shear acting on the slab critical section as defined by ACI 318, and V_o is the direct punching shear strength as defined by ACI 318.
[d] "Yes" shall be used where the area of effectively continuous main bottom bars passing through the column cage in each direction is greater than or equal to $0.5V_g/(\phi f_y)$. Where the slab is post-tensioned, "Yes" shall be used where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, "No" shall be used.
[e] Action shall be treated as force controlled.

Table 10-16. Numerical Acceptance Criteria for Linear Procedures—Two-Way Slabs and Slab–Column Connections

		m-Factors[a]				
		Performance Level				
			Component Type			
			Primary		Secondary	
Conditions		**IO**	**LS**	**CP**	**LS**	**CP**
Condition i. Reinforced concrete slab–column connections[b]						
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]					
0	Yes	2	2.75	3.5	3.5	4.5
0.2	Yes	1.5	2.5	3	3	3.75
0.4	Yes	1	2	2.25	2.25	3
≥0.6	Yes	1	1	1	1	2.25
0	No	2	2.25	2.25	2.25	2.75
0.2	No	1.5	2	2	2	2.25
0.4	No	1	1.5	1.5	1.5	1.75
0.6	No	1	1	1	1	1
>0.6	No	—[e]	—[e]	—[e]	—[e]	—[e]
Condition ii. Post-tensioned slab–column connections[b]						
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]					
0	Yes	1.5	2	2.5	2.5	3.25
0.6	Yes	1	1	1	2	2.25
>0.6	Yes	1	1	1	1.5	1.75
0	No	1.25	1.75	1.75	1.75	2
0.6	No	1	1	1	1	1
>0.6	No	—[e]	—[e]	—[e]	—[e]	—[e]
Condition iii. Slabs controlled by inadequate development or splicing along the span[b]						
		—[e]	—[e]	—[e]	3	4
Condition iv. Slabs controlled by inadequate embedment into slab–column joint[b]						
		2	2	3	3	4

[a] Values between those listed in the table shall be determined by linear interpolation.
[b] Where more than one of conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.
[c] V_g is the gravity shear acting on the slab critical section as defined by ACI 318, and V_o is the direct punching shear strength as defined by ACI 318.
[d] "Yes" should be used where the area of effectively continuous main bottom bars passing through the column cage in each direction is greater than or equal to $0.5V_g/(\phi f_y)$. Where the slab is post-tensioned, "Yes" shall be used where at least one of the posttensioning tendons in each direction passes through the column cage. Otherwise, "No" should be used.
[e] Action shall be treated as force controlled.

1. Moments, shears, torsions, and development and splice actions corresponding to the development of component strength in slabs and columns; and
2. Axial load in columns, considering likely plastic action in components above the level in question.

Design actions shall be compared with design strengths in accordance with Section 7.5.2.2, and *m*-factors for slab–column frame components should be selected from Tables 10-9 and 10-16.

Where the average DCR for columns at a level exceeds the average value for slabs at the same level and exceeds the greater of 1.0 and *m*/2, the element shall be defined as a weak story element and shall be evaluated by the procedure for weak story elements in Section 10.4.2.4.1.

10.4.4.4.2 Nonlinear Static and Dynamic Procedures. Inelastic response shall be restricted to actions in Tables 10-8 and 10-15, except where it is demonstrated by experimental evidence and analysis that other inelastic actions are acceptable for the selected Performance Levels. Other actions shall be defined as force controlled.

Calculated component actions shall satisfy the requirements of Section 7.5.3.2. Maximum permissible inelastic deformations shall be taken from Tables 10-8 and 10-15. Alternative values shall be permitted where justified by experimental evidence and analysis.

10.4.4.5 Retrofit Measures for Slab–Column Moment Frames. Seismic retrofit measures for slab–column moment frames shall meet the requirements of Section 10.3.7 and other provisions of this standard.

10.5 PRECAST CONCRETE FRAMES

10.5.1 Types of Precast Concrete Frames. Precast concrete frames shall be defined as those elements that are constructed from individually made beams and columns that are assembled to create gravity-load-carrying systems. These systems shall include those that are considered in design to resist seismic forces and those that are considered in design as secondary elements that do not resist seismic forces but must resist the effects of deformations resulting from seismic forces.

10.5.1.1 Precast Concrete Frames Expected to Resist Seismic Forces. Frames of this classification shall be assembled using either reinforcement and wet concrete or dry joints (connections are made by bolting, welding, posttensioning, or other similar means) in a way that results in significant seismic-force resistance in the framing element. Frames of this classification resist seismic forces either acting alone or acting in conjunction with structural walls, braced frames, or other seismic-force-resisting elements.

10.5.1.2 Precast Concrete Frames Not Expected to Resist Seismic Forces Directly. Frames of this classification shall be assembled using dry joints in a way that does not result in significant seismic-force resistance in the framing element. Structural walls, braced frames, or moment frames provide the entire seismic-force resistance, with the precast concrete frame system deforming in a manner that is compatible with the structure as a whole.

10.5.2 Precast Concrete Frames Expected to Resist Seismic Forces

10.5.2.1 General. The analytical model for a frame element of this classification shall represent strength, stiffness, and deformation capacity of beams, columns, beam–column joints, and other components of the frame. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other components, including nonstructural components, shall be included. All other considerations of Section 10.4.2.1 shall be taken into account. In addition, the effects of shortening caused by creep, and other effects of prestressing and posttensioning on member behavior, shall be evaluated. Where dry joints are used in assembling the precast system, consideration shall be given to the effect of those joints on overall behavior. Where connections yield under the specified seismic forces, the analysis model shall take this effect into account.

10.5.2.2 Stiffness of Precast Concrete Frames Expected to Resist Seismic Forces. Stiffness for analysis shall be as defined in Section 10.4.2.2. The effects of prestressing shall be considered where computing the effective stiffness values using Table 10-5. Flexibilities associated with connections shall be included in the analytical model.

10.5.2.3 Strength of Precast Concrete Frames Expected to Resist Seismic Forces. Component strength shall be computed according to the requirements of Section 10.4.2.3, with the additional requirement that the following effects be included in the analysis:

1. Effects of prestressing that are present, including but not limited to reduction in rotation capacity, secondary stresses induced, and amount of effective prestress force remaining;
2. Effects of construction sequence, including the possibility of construction of the moment connections occurring after portions of the structure are subjected to dead loads;
3. Effects of restraint caused by interaction with interconnected wall or brace components; and
4. Effects of connection strength, considered in accordance with Section 10.3.6.

10.5.2.4 Acceptance Criteria for Precast Concrete Frames Expected to Resist Seismic Forces. Acceptance criteria for precast concrete frames expected to resist seismic forces shall be as specified in Section 10.4.2.4, except that the factors defined in Section 10.4.2.3 shall also be considered. Connections shall comply with the requirements of Section 10.3.6.

10.5.2.5 Retrofit Measures for Precast Concrete Frames Expected to Resist Seismic Forces. Seismic retrofit measures for precast concrete frames shall meet the requirements of Section 10.3.7 and other provisions of this standard.

10.5.3 Precast Concrete Frames Not Expected to Resist Seismic Forces Directly

10.5.3.1 General. The analytical model for precast concrete frames that are not expected to resist seismic forces directly shall comply with the requirements of Section 10.5.2.1 and shall include the effects of deformations that are calculated to occur under the specified seismic loadings.

10.5.3.2 Stiffness of Precast Concrete Frames Not Expected to Resist Seismic Forces Directly. The analytical model shall include either realistic lateral stiffness of these frames to evaluate the effects of deformations under seismic forces or, if the lateral stiffness is ignored in the analytical model, the effects of calculated building drift on these frames shall be evaluated separately. The analytical model shall consider the negative effects of connection stiffness on component response where that stiffness results in actions that can cause component failure.

10.5.3.3 Strength of Precast Concrete Frames Not Expected to Resist Seismic Forces Directly. Component strength shall be computed according to the requirements of Section 10.5.2.3. All components shall have sufficient strength and ductility to transmit induced forces from one member to another and to the designated seismic-force-resisting system.

10.5.3.4 Acceptance Criteria for Precast Concrete Frames Not Expected to Resist Seismic Forces Directly. Acceptance criteria for components in precast concrete frames not expected to resist seismic forces directly shall be as specified in Section 10.5.2.4. All moments, shear forces, and axial loads induced through the deformation of the structural system shall be checked using appropriate criteria in the referenced section.

10.5.3.5 Retrofit Measures for Precast Concrete Frames Not Expected to Resist Seismic Forces Directly. Seismic retrofit measures for precast moment frames shall meet the requirements of Section 10.3.7 and other provisions of this standard.

10.6 CONCRETE FRAMES WITH INFILLS

10.6.1 Types of Concrete Frames with Infills. Concrete frames with infills consist of complete gravity-load-carrying concrete frames infilled with masonry or concrete, constructed in such a way that the infill and the concrete frame interact when subjected to vertical and seismic forces.

Isolated infills are infills isolated from the surrounding frame complying with the minimum gap requirements specified in Section 11.4.1. If all infills in a frame are isolated infills, the frame shall be analyzed as an isolated frame according to provisions given elsewhere in this chapter, and the isolated infill panels shall be analyzed according to the requirements of Chapter 11.

10.6.1.1 Types of Frames. The provisions of Section 10.6 shall apply to concrete frames, as defined in Sections 10.4, 10.5, and 10.9, which interact with infills.

10.6.1.2 Masonry Infills. The provisions of Section 10.6 shall apply to masonry infills, as defined in Chapter 11, which interact with concrete frames.

10.6.1.3 Concrete Infills. The provisions of Section 10.6 shall apply to concrete infills that interact with concrete frames, where the infills were constructed to fill the space within the bay of a complete gravity frame without special provision for continuity from story to story. The concrete of the infill shall be evaluated separately from the concrete of the frame.

10.6.2 Concrete Frames with Masonry Infills

10.6.2.1 General. The analytical model for a concrete frame with masonry infills shall represent strength, stiffness, and deformation capacity of beams, slabs, columns, beam–column joints, masonry infills, and all connections and components of the element. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with nonstructural components shall be included.

For a concrete frame with masonry infill resisting seismic forces within its plane, modeling of the response using a linear elastic model shall be permitted provided that the infill does not crack when subjected to design seismic forces. If the infill does not crack when subjected to design seismic forces, modeling the assemblage of frame and infill as a homogeneous medium shall be permitted.

For a concrete frame with masonry infills that cracks when subjected to design seismic forces, modeling of the response using a diagonally braced frame model, in which the columns act as vertical chords, the beams act as horizontal ties, and the infill acts as an equivalent compression strut, shall be permitted. Requirements for the equivalent compression strut analogy shall be as specified in Chapter 11.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill, as specified in Chapter 11. In frames with full-height masonry infills, the evaluation shall include the effect of strut compression forces applied to the column and beam, eccentric from the beam–column joint. In frames with partial-height masonry infills, the evaluation shall include the reduced effective length of the columns above the infilled portion of the bay.

10.6.2.2 Stiffness of Concrete Frames with Masonry Infills

10.6.2.2.1 Linear Static and Dynamic Procedures. In frames that have infills in some bays and no infill in other bays, the restraint of the infill shall be represented as described in Section 10.6.2.1, and the noninfilled bays shall be modeled as frames as specified in appropriate portions of Sections 10.4, 10.5, and 10.9. Where infills create a discontinuous wall, the effects of the discontinuity on overall building performance shall be evaluated. Effective stiffnesses shall be in accordance with Section 10.3.1.2.

10.6.2.2.2 Nonlinear Static Procedure. Nonlinear load-deformation relations for use in analysis by the NSP shall follow the requirements of Section 10.3.1.2.2.

Modeling beams and columns using nonlinear truss elements shall be permitted in infilled portions of the frame. Beams and columns in noninfilled portions of the frame shall be modeled using the relevant specifications of Sections 10.4, 10.5, and 10.9. The model shall be capable of representing inelastic response along the component lengths.

Monotonic load-deformation relations shall be according to the generalized relation shown in Fig. 10-1, except that different relations shall be permitted where verified by tests. Numerical quantities in Fig. 10-1 shall be derived from tests or by analytical procedures, as specified in Chapter 7, and shall take into account the interactions between frame and infill components. Alternatively, the following procedure shall be permitted for monolithic reinforced concrete frames.

1. For beams and columns in noninfilled portions of frames, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, the plastic hinge rotation capacities shall be as defined by Tables 10-7 and 10-8.
2. For masonry infills, the generalized deformations and control points shall be as defined in Chapter 11.
3. For beams and columns in infilled portions of frames, where the generalized deformation is taken as elongation or compression displacement of the beams or columns, the tension and compression strain capacities shall be as specified in Table 10-17.

10.6.2.2.3 Nonlinear Dynamic Procedure. Nonlinear load–deformation relations for use in analysis by NDP shall model the complete hysteretic behavior of each component using properties verified by tests. Unloading and reloading properties shall represent stiffness and strength degradation characteristics.

10.6.2.3 Strength of Concrete Frames with Masonry Infills. Strengths of reinforced concrete components shall be calculated according to the general requirements of Section 10.3.2, as modified by other specifications of this chapter. Strengths of masonry infills shall be calculated according to the requirements of Chapter 11. Strength calculations shall consider the following:

1. Limitations imposed by beams, columns, and joints in noninfilled portions of frames;
2. Tensile and compressive capacity of columns acting as boundary components of infilled frames;
3. Local forces applied from the infill to the frame;
4. Strength of the infill; and
5. Connections with adjacent components.

10.6.2.4 Acceptance Criteria for Concrete Frames with Masonry Infills

10.6.2.4.1 Linear Static and Dynamic Procedures. All component actions shall be classified as either deformation controlled or force controlled, as defined in Section 7.5.1. In primary components, deformation-controlled actions shall be restricted to flexure and axial actions in beams, slabs, and columns, and lateral deformations in masonry infill panels. In secondary components, deformation-controlled actions shall be restricted to those actions identified for the isolated frame in Sections 10.4, 10.5, and 10.9, as appropriate, and for the masonry infill in Section 11.4.

Design actions shall be determined as prescribed in Chapter 7. Where calculated DCR values exceed unity, the following design actions shall be determined using limit analysis principles as prescribed in Chapter 7: (1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams, columns, or masonry infills; and (2) column axial load corresponding to development of the flexural capacity of the infilled frame acting as a cantilever wall.

Design actions shall be compared with strengths in accordance with Section 7.5.2.2.

Values of m-factors shall be as specified in Section 11.4.2.4 for masonry infills; applicable portions of Sections 10.4, 10.5, and 10.9 for concrete frames; and Table 10-18 for columns modeled as tension and compression chords. Those components that have

Table 10-17. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Infilled Frames

Conditions	Modeling Parameters[a]			Acceptance Criteria		
				Total Strain		
	Total Strain		Residual Strength Ratio		Performance Level	
	d	*e*	*c*	IO	LS	CP
i. Columns modeled as compression chords[b]						
Columns confined along entire length[c]	0.02	0.04	0.4	0.003	0.03	0.04
All other cases	0.003	0.01	0.2	0.002	0.01	0.01
ii. Columns modeled as tension chords[b]						
Columns with well-confined splices or no splices	0.05	0.05	0.0	0.01	0.04	0.05
All other cases	See note *d*	0.03	0.2	See note *d*	0.02	0.03

[a] Interpolation shall not be permitted.
[b] If load reversals result in both conditions i and ii applying to a single column, both conditions shall be checked.
[c] A column shall be permitted to be considered to be confined along its entire length where the quantity of hoops along the entire story height including the joint is equal to three-quarters of that required by ACI 318 for boundary components of concrete structural walls. The maximum longitudinal spacing of sets of hoops shall not exceed either $h/3$ or $8d_b$.
[d] Potential for splice failure shall be evaluated directly to determine the modeling and acceptance criteria. For these cases, refer to the generalized procedure of Section 10.6.3.2.

Table 10-18. Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Infilled Frames

Conditions	*m*-Factors[a]				
	Performance Level				
		Component Type			
		Primary		Secondary	
	IO	LS	CP	LS	CP
i. Columns modeled as compression chords[b]					
Columns confined along entire length[c]	1	3	4	4	5
All other cases	1	1	1	1	1
ii. Columns modeled as tension chords[b]					
Columns with well-confined splices or no splices	3	4	5	5	6
All other cases	1	2	2	3	4

[a] Interpolation shall not be permitted.
[b] If load reversals result in both conditions i and ii applying to a single column, both conditions shall be checked.
[c] A column can be considered to be confined along its entire length where the quantity of hoops along the entire story height, including the joint, is equal to three-quarters of that required by ACI 318 for boundary components of concrete structural walls. The maximum longitudinal spacing of sets of hoops shall not exceed either $h/3$ or $8d_b$.

design actions less than strengths shall be assumed to satisfy the performance criteria for those components.

10.6.2.4.2 Nonlinear Static and Dynamic Procedures. In the design model, inelastic response shall be restricted to those components and actions that are permitted for isolated frames as specified in Sections 10.4, 10.5, and 10.9, and for masonry infills as specified in Section 11.4.

Calculated component actions shall satisfy the requirements of Section 7.5.3.2 and shall not exceed the numerical values listed in Table 10-17; the relevant tables for isolated frames given in Sections 10.4, 10.5, and 10.9; and the relevant tables for masonry infills given in Chapter 11. Component actions not listed in Tables 10-7, 10-8, and 10-10 shall be treated as force controlled. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

10.6.2.5 Retrofit Measures for Concrete Frames with Masonry Infills. Seismic retrofit measures for concrete frames with masonry infills shall meet the requirements of Section 10.3.7 and other provisions of this standard.

10.6.3 Concrete Frames with Concrete Infills

10.6.3.1 General. The analytical model for a concrete frame with concrete infills shall represent the strength, stiffness, and deformation capacity of beams, slabs, columns, beam–column joints, concrete infills, and all connections and components of the elements. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with nonstructural components shall be included.

The analytical model shall be established considering the relative stiffness and strength of the frame and the infill, as well as the level of deformations and associated damage. For low deformation levels, and for cases where the frame is relatively flexible, the infilled frame shall be permitted to be modeled as a shear wall, with openings modeled where they occur. In other cases, the frame–infill system shall be permitted to be modeled using a braced-frame analogy such as that described for concrete frames with masonry infills in Section 10.6.2.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill as specified in Chapter 11. In frames with full-height infills, the evaluation shall include the effect of strut compression forces applied to the column and beam eccentric from the beam–column joint. In frames with partial-height infills, the evaluation shall include the reduced effective length of the columns above the infilled portion of the bay.

In frames that have infills in some bays and no infills in other bays, the restraint of the infill shall be represented as described in this section, and the noninfilled bays shall be modeled as frames as specified in appropriate portions of Sections 10.4, 10.5, and 10.9. Where infills create a discontinuous wall, the effects of the discontinuity on overall building performance shall be evaluated.

10.6.3.2 Stiffness of Concrete Frames with Concrete Infills

10.6.3.2.1 Linear Static and Dynamic Procedures. Effective stiffnesses shall be calculated according to the principles of Section 10.3.1.2.1 and the procedure of Section 10.6.2.2.1.

10.6.3.2.2 Nonlinear Static Procedure. Nonlinear load–deformation relations for use in analysis by NSP shall follow the requirements of Section 10.3.1.2.2.

Monotonic load–deformation relations shall be according to the generalized relation shown in Fig. 10-1, except that different relations shall be permitted where verified by tests. Numerical quantities in Fig. 10-1 shall be derived from tests or by analysis procedures specified in Section 7.6 and shall take into account the interactions between frame and infill components. Alternatively, the procedure of Section 10.6.2.2.2 shall be permitted for the development of nonlinear modeling parameters for concrete frames with concrete infills.

10.6.3.2.3 Nonlinear Dynamic Procedure. Nonlinear load–deformation relations for use in analysis by NDP shall model the complete hysteretic behavior of each component using properties verified by tests. Unloading and reloading properties shall represent stiffness and strength degradation characteristics.

10.6.3.3 Strength of Concrete Frames with Concrete Infills. Strengths of reinforced concrete components shall be calculated according to the general requirements of Section 10.4.2, as modified by other specifications of this chapter. Strength calculations shall consider the following:

1. Limitations imposed by beams, columns, and joints in unfilled portions of frames;
2. Tensile and compressive capacity of columns acting as boundary components of infilled frames;
3. Local forces applied from the infill to the frame;
4. Strength of the infill; and
5. Connections with adjacent components.

Strengths of existing concrete infills shall be determined considering shear strength of the infill panel. For this calculation, procedures specified in Section 10.7.2.3 shall be used for calculation of the shear strength of a wall segment.

Where the frame and concrete infill are assumed to act as a monolithic wall, flexural strength shall be based on continuity of vertical reinforcement in both (1) the columns acting as boundary components and (2) the infill wall, including anchorage of the infill reinforcement in the boundary frame.

10.6.3.4 Acceptance Criteria for Concrete Frames with Concrete Infills. The acceptance criteria for concrete frames with concrete infills shall comply with relevant acceptance criteria of Sections 10.6.2.4, 10.7, and 10.8.

10.6.3.5 Retrofit Measures for Concrete Frames with Concrete Infills. Seismic retrofit measures for concrete frames with concrete infills shall meet the requirements of Section 10.3.7 and other provisions of this standard.

10.7 CONCRETE STRUCTURAL WALLS

10.7.1 Types of Concrete Structural Walls and Associated Components. The provisions of Section 10.7 shall apply to all reinforced concrete structural walls in all types of structural systems that incorporate reinforced concrete structural walls. This set of types includes isolated structural walls, structural walls used in wall-frame systems, coupled structural walls, and discontinuous structural walls. Structural walls shall be permitted to be considered as solid walls if they have openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated structural walls shall be defined as walls that have a regular pattern of openings in both horizontal and vertical directions that creates a series of wall pier and deep beam components referred to as wall segments.

Coupling beams shall comply with provisions of Section 10.7.2 and shall be exempted from the provisions for beams covered in Section 10.4.

10.7.1.1 Monolithic Reinforced Concrete Structural Walls and Wall Segments. Monolithic reinforced concrete structural walls shall consist of vertical cast-in-place elements, either uncoupled or coupled, in open or closed shapes. These walls shall have relatively continuous cross sections and reinforcement and shall provide both vertical and lateral force resistance, in contrast with infilled walls defined in Section 10.6.1.3.

Structural walls or wall segments with axial loads greater than 0.35 P_o shall not be considered effective in resisting seismic forces. For the purpose of determining effectiveness of structural walls or wall segments, the use of axial loads based on limit-state analysis shall be permitted.

10.7.1.2 Reinforced Concrete Columns Supporting Discontinuous Structural Walls. Reinforced concrete columns supporting discontinuous structural walls shall be analyzed in accordance with the requirements of Section 10.4.2.

10.7.1.3 Reinforced Concrete Coupling Beams. Reinforced concrete coupling beams used to link two structural walls together shall be evaluated and rehabilitated to comply with the requirements of Section 10.7.2.

10.7.2 Reinforced Concrete Structural Walls, Wall Segments, and Coupling Beams

10.7.2.1 General. The analytical model for a structural wall element shall represent the stiffness, strength, and deformation capacity of the shear wall. Potential failure in flexure, shear, and reinforcement development at any point in the structural wall shall be considered. Interaction with other structural and nonstructural components shall be included.

Slender structural walls and wall segments shall be permitted to be modeled as equivalent beam–column elements that include both flexural and shear deformations. The flexural strength of beam–column elements shall include the interaction of axial load and bending. The rigid connection zone at beam connections to this equivalent beam–column element shall represent the distance from the wall centroid to the edge of the wall. Unsymmetrical wall sections shall be modeled with the different bending capacities for the two loading directions.

A beam element that incorporates both bending and shear deformations shall be used to model coupling beams. The element inelastic response shall account for the loss of shear strength and stiffness during reversed cyclic loading to large deformations. For coupling beams that have diagonal reinforcement satisfying ACI 318 requirements, a beam element representing flexure only shall be permitted.

The diaphragm action of concrete slabs that interconnect structural walls and frame columns shall be represented in the model.

10.7.2.2 Stiffness of Reinforced Concrete Structural Walls, Wall Segments, and Coupling Beams. The effective stiffness of all the elements discussed in Section 10.7 shall be defined based on the material properties, component dimensions, reinforcement quantities, boundary conditions, and current state of the member with respect to cracking and stress levels. Alternatively, use of values for effective stiffness given in Table 10-5 shall be permitted.

For coupling beams, the effective stiffness values given in Table 10-5 for nonprestressed beams shall be used unless alternative stiffnesses are determined by more detailed analysis.

10.7.2.2.1 Linear Static and Dynamic Procedures. Structural walls and associated components shall be modeled considering axial, flexural, and shear stiffness. For closed and open wall shapes, such as box, T, L, I, and C sections, the effective tension or compression flange widths shall be as specified in Section 10.3.1.3. The calculated stiffnesses to be used in analysis shall be in accordance with the requirements of Section 10.3.1.2.

Joints between structural walls and frame elements shall be modeled as stiff components or rigid components, as appropriate.

10.7.2.2.2 Nonlinear Static Procedure. Nonlinear load-deformation relations for use in analysis by nonlinear static and dynamic procedures shall comply with the requirements of Section 10.3.1.2.

Monotonic load–deformation relationships for analytical models that represent structural walls, wall elements, and coupling beams shall be in accordance with the generalized relation shown in Fig. 10-1.

For structural walls and wall segments that have inelastic behavior under lateral loading that is governed by flexure, the following approach shall be permitted. The load–deformation relationship in Fig. 10-1 shall be used with the x-axis of Fig. 10-1 taken as the rotation over the plastic hinging region at the end of the member shown in Fig. 10-4. The hinge rotation at point B in

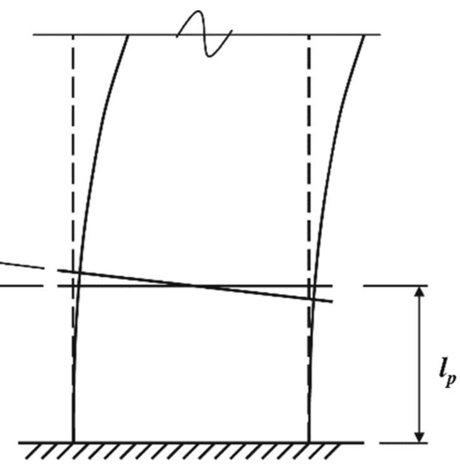

Figure 10-4. Plastic Hinge Rotation in Shear Wall Where Flexure Dominates Inelastic Response

Fig. 10-1 corresponds to the yield point, θ_y, and shall be calculated in accordance with Eq. (10-5):

$$\theta_{yE} = \left(\frac{M_{yE}}{(EI)_{eff}}\right) l_p \tag{10-5}$$

where l_p = Assumed plastic hinge length.

For analytical models of structural walls and wall segments, the value of l_p shall be set equal to 0.5 times the flexural depth of the element but less than one story height for structural walls and less than 50% of the element length for wall segments.

Values for the variables *a, b*, and *c* required to define the location of points C, D, and E in Fig. 10-1(a) shall be as specified in Table 10-19.

For structural walls and wall segments whose inelastic response is controlled by shear, the following approach shall be permitted. The load–deformation relationship in Fig. 10-1(c) shall be used, with the x-axis of Fig. 10-1(c) taken as the lateral drift ratio. Alternatively, the load–deformation relationship in Fig. 10-1(b) shall be permitted, with the x-axis of Fig. 10-1(b) taken as the lateral drift ratio. For structural walls, this drift shall be the story drift, as shown in Fig. 10-5. For wall segments, Fig. 10-5 shall represent the member drift.

For coupling beams, the following approach shall be permitted. The load–deformation relationship in Fig. 10-1(b) shall be used, with the x-axis of Fig. 10-1(b) taken as the chord rotation, as defined in Fig. 10-6.

Values for the variables *d, e, f, g*, and *c,* required to find the points B, C, D, E, and F in Fig. 10-1(b) or 10-1(c), shall be as specified in Table 10-20 for the appropriate members. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

10.7.2.2.3 Nonlinear Dynamic Procedure. For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. Use of the generalized load–deformation relation shown in Fig. 10-1 to represent the envelope relation for the analysis shall be permitted. The unloading and reloading stiffnesses and strengths, and any pinching of the load-versus-rotation hysteresis loops, shall reflect the behavior experimentally observed for wall elements similar to the one under investigation.

10.7.2.3 Strength of Reinforced Concrete Structural Walls, Wall Segments, and Coupling Beams. Component strengths

Table 10-19. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Structural Walls and Associated Components Controlled by Flexure

Conditions			Plastic Hinge Rotation (radians)		Residual Strength Ratio	Acceptable Plastic Hinge Rotation[a] (radians)		
						Performance Level		
			a	b	c	IO	LS	CP
i. Structural walls and wall segments								
$\frac{(A_s - A'_s)f_{yE} + P}{t_w l_w f'_{cE}}$	$\frac{V}{t_w l_w \sqrt{f'_{cE}}}$	Confined Boundary[b]						
≤0.1	≤4	Yes	0.015	0.020	0.75	0.005	0.015	0.020
≤0.1	≥6	Yes	0.010	0.015	0.40	0.004	0.010	0.015
≥0.25	≤4	Yes	0.009	0.012	0.60	0.003	0.009	0.012
≥0.25	≥6	Yes	0.005	0.010	0.30	0.0015	0.005	0.010
≤0.1	≤4	No	0.008	0.015	0.60	0.002	0.008	0.015
≤0.1	≥6	No	0.006	0.010	0.30	0.002	0.006	0.010
≥0.25	≤4	No	0.003	0.005	0.25	0.001	0.003	0.005
≥0.25	≥6	No	0.002	0.004	0.20	0.001	0.002	0.004
ii. Structural wall coupling beams[c]								
Longitudinal reinforcement and transverse reinforcement[d]		$\frac{V}{t_w l_w \sqrt{f'_{cE}}}$	d	e	c			
Nonprestressed longitudinal reinforcement with conforming transverse reinforcement		≤3	0.025	0.050	0.75	0.010	0.025	0.050
		≥6	0.020	0.040	0.50	0.005	0.020	0.040
Nonprestressed longitudinal reinforcement with nonconforming transverse reinforcement		≤3	0.020	0.035	0.50	0.006	0.020	0.035
		≥6	0.010	0.025	0.25	0.005	0.010	0.025
Diagonal reinforcement		NA	0.030	0.050	0.80	0.006	0.030	0.050

[a] Linear interpolation between values listed in the table shall be permitted.
[b] A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. Otherwise, boundary elements shall be considered not confined.
[c] For coupling beams spanning 8 ft 0 in., with bottom reinforcement continuous into the supporting walls, acceptance criteria values shall be permitted to be doubled for LS and CP performance.
[d] Nonprestressed longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (a) closed stirrups over the entire length of the coupling beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.

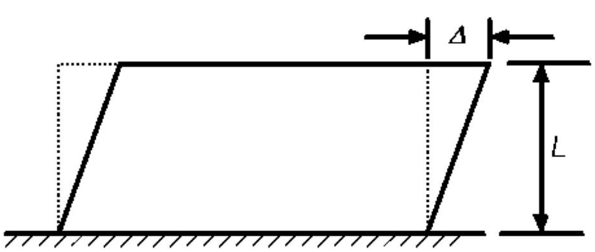

Figure 10-5. Story Drift in Structural Wall Where Shear Dominates Inelastic Response

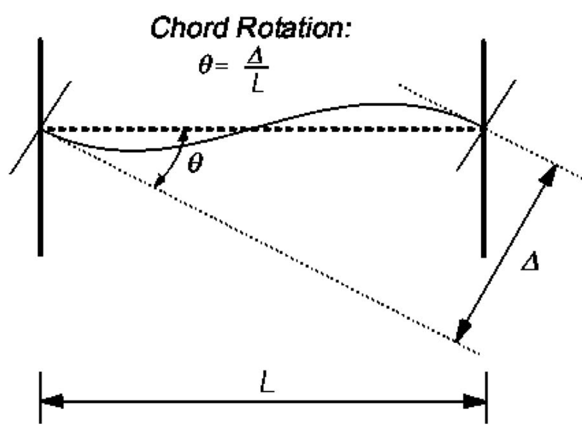

Figure 10-6. Chord Rotation for Structural Wall Coupling Beams